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Reshaping Breakwaters

on the stability of roundheads and trunk erosion in oblique waves

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Publication date:
1987

Document Version
Også kaldet Forlagets PDF

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Citation for published version (APA):

Burcharth, H. F., & Frigaard, P. (1987). *Reshaping Breakwaters: on the stability of roundheads and trunk erosion in oblique waves*. Aalborg Universitetscenter, Inst. for Vand, Jord og Miljøteknik, Laboratoriet for Hydraulik og Havnebygning.

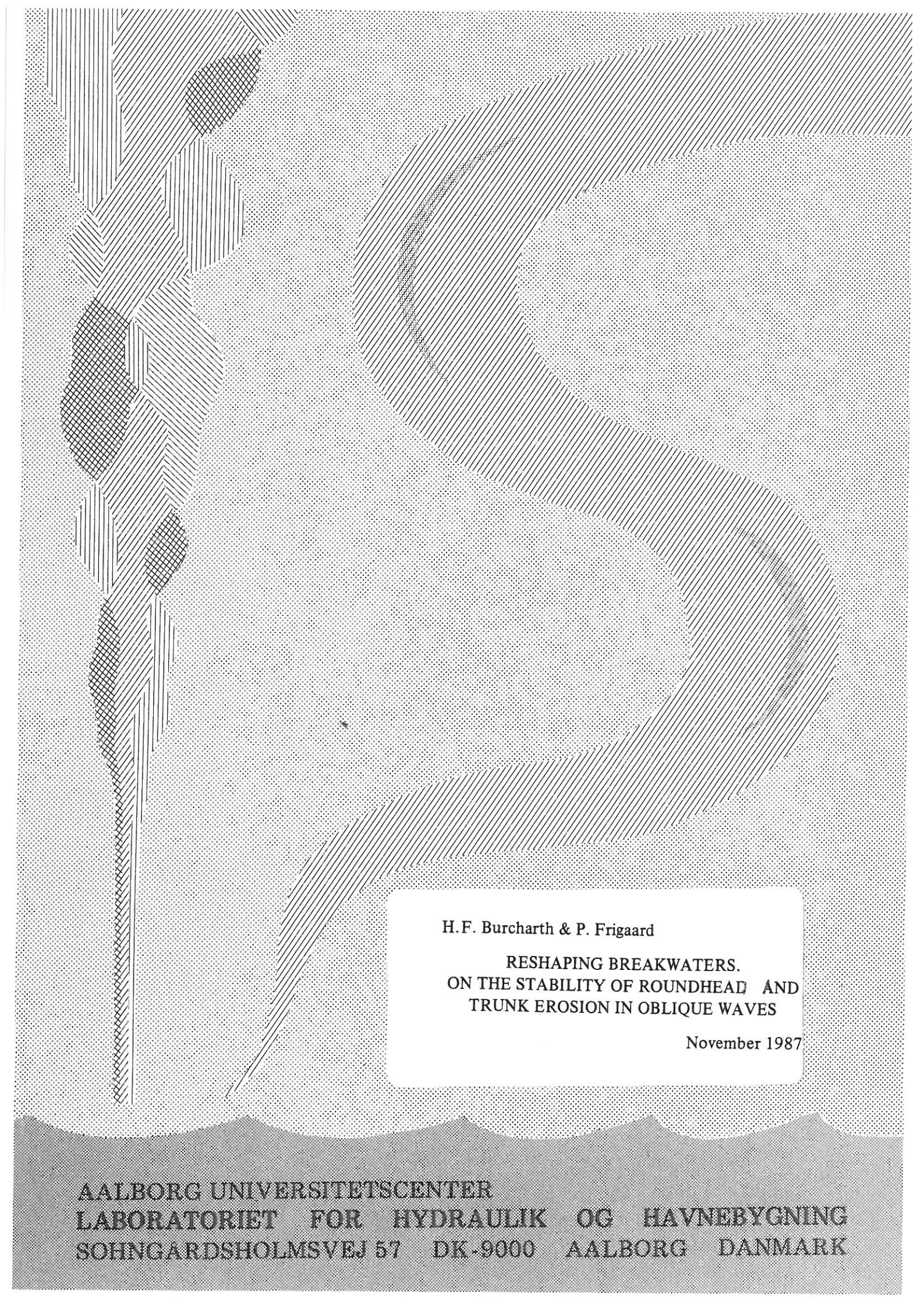
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RESHAPING BREAKWATERS.
ON THE STABILITY OF ROUNDHEAD AND
TRUNK EROSION IN OBLIQUE WAVES

November 1987

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Presented at

The Berm Type Breakwater Conference, Ottawa, Canada, September 1987.

**RESHAPING BREAKWATERS.
ON THE STABILITY OF
ROUNDHEADS AND TRUNK EROSION IN OBLIQUE WAVES**

by

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INTRODUCTION

The paper deals with the 3-dimensional stability of the type of rubble mound breakwaters where reshaping of the mound due to wave action is foreseen in the design. Such breakwaters are commonly named sacrificial types and berm types. The latter is due to the relatively large volume of armour stones placed in a seaward berm. However, as also conventional armoured breakwaters some times do contain a berm it is assumed that a better and more ambiguous designation would be "reshaping" rubble mound breakwaters.

The stability of a reshaping type breakwater was tested in a 3-dimensional model at The Hydraulics Laboratory, Department of Civil Engineering, University of Aalborg.

The object was to study the stability/erosion of the breakwater head and the trunk, the latter exposed to both head-on and oblique irregular waves.

To avoid too many parameters a simple breakwater geometry and only one class of stones were used.

MODEL TEST SET-UP

The stone material

The model consisted of one grading of crushed stones with a density $\rho_s = 2.65 \text{ t/m}^3$ (metric ton) and a gradation as given in Figs. 1 and 2.

It was found that the relationship between the sieve diameter (quadratic sieve) d and the stone volume V and stone weight w is

$$V = 0.7d^3 = \frac{w}{\rho_s}$$

d is regarded a characteristic diameter of the stones.

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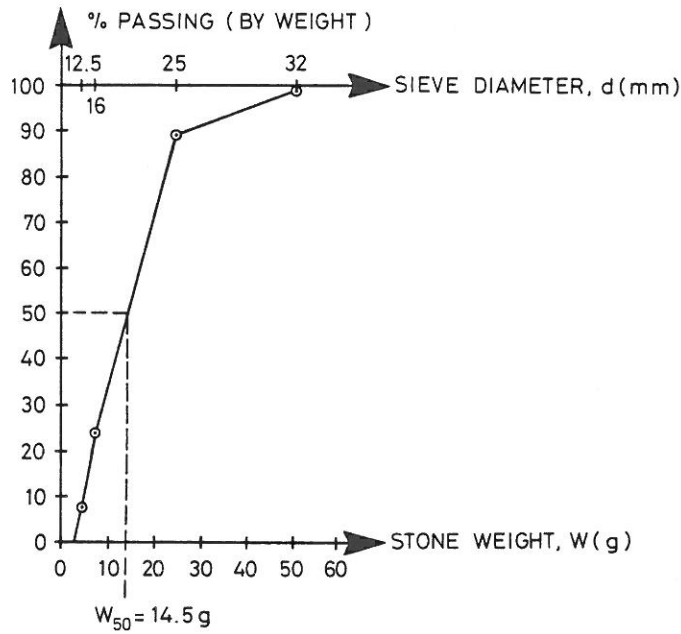


Fig. 1. Gradation of model stones (linear representation).

As seen from Fig. 1 w_{50} is found to be 14.5 g. However, it is most likely that an extra point on the gradation curve in the sieve interval 16-25 mm would have shifted the graph to the left and thereby given a w_{50} smaller than 14.5 g.

This is investigated by the log representation, Fig. 2.

The stone weight corresponding to $d_{50}^{\log} = 19 \text{ mm}$ is $w_{d_{50}} = \rho_s \cdot 0.7 d^3 = 2.65 \cdot 0.7 \cdot 1.9^3 = 12.7 \text{ g} \geq w_{50}^{\log} = 12 \text{ g}$.

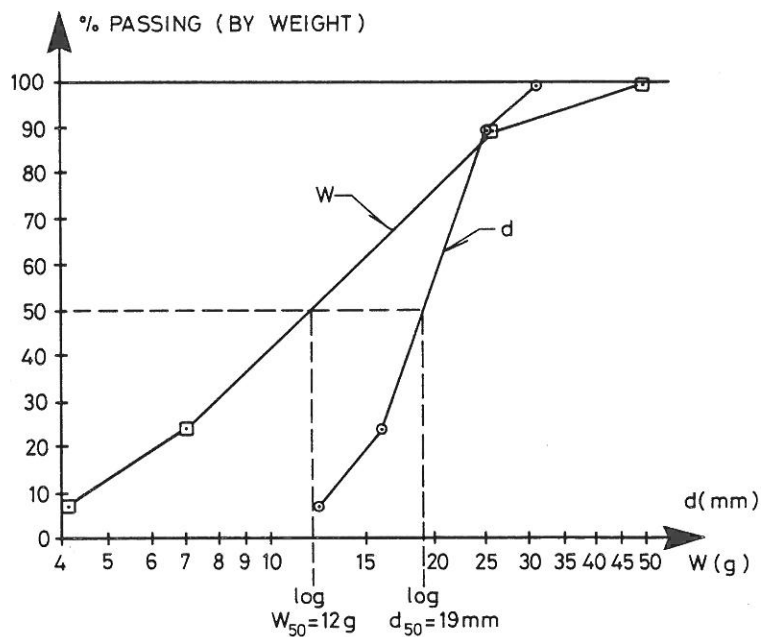


Fig. 2. Gradation of model stones (log - linear representation).

The log representations confirm that w_{50} is somewhat less than 14.5 g.

Based on the various figures a w_{50} of 12.7 g corresponding to $d_{50}^{\log} = 19$ mm is chosen as the most correct value.

It should be noted that for the investigation of longshore transport in oblique waves samples of stones without diameters less than 16 mm were used. For these samples $d_{50}^{\log} = 22$ mm and $w_{d_{50}} = 20.3$ g.

The geometry of the model

Fig. 3 shows the cross section of the model (before each test) and the lay-out of the model in the wave basin.

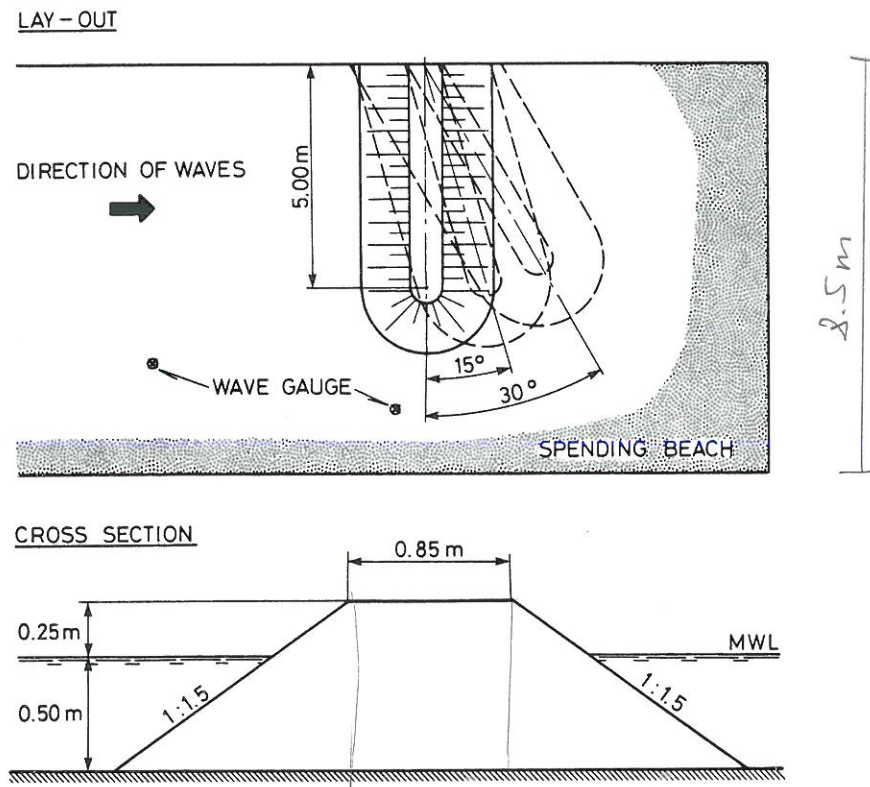


Fig. 3. Lay-out and cross section of the model.

Waves

All waves were irregular waves generated in accordance with a random phase JONSWAP-type spectrum (peakedness parameter $\gamma = 3.3$ and width parameters $\sigma_f = 0.10$ for $f \leq f_p$ and $\sigma_f = 0.50$ for $f > f_p$).

The tested sea states are specified in Fig. 4.

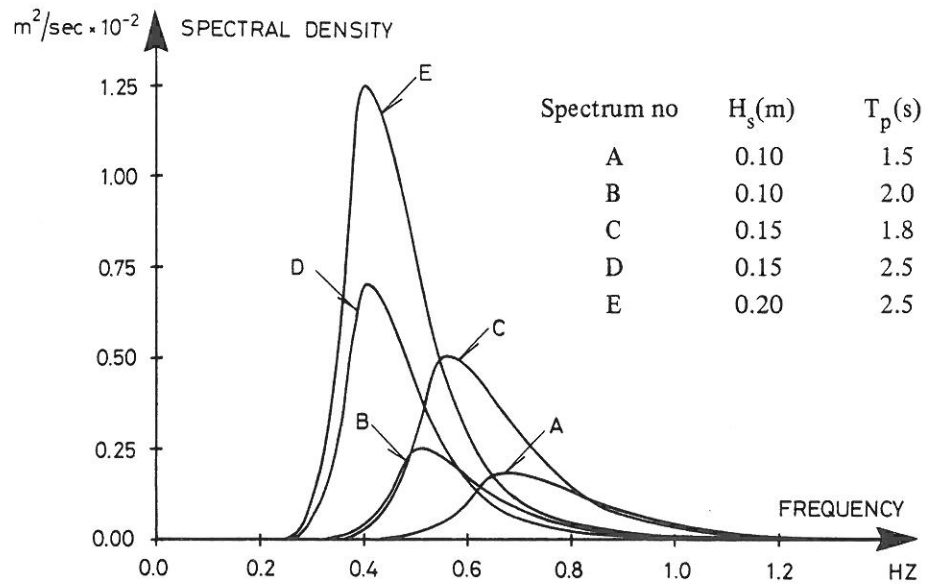


Fig. 4. Wave spectra.

The following three angles of attack were tested: $\alpha = 0$ (head-on waves), $\alpha = 15^\circ$, and $\alpha = 30^\circ$.

MODEL CONSIDERATIONS

The sea states were chosen in the range from very little erosion to fast erosion of the profile under oblique wave attack.

An indication of the relative stability of the profiles can be given by the dimensionless parameter $H_s/\Delta \cdot D_{n50}$, where $\Delta = \rho_s/\rho - 1$ and D_{n50} is a nominal diameter defined as $(w_{50}/\rho_s)^{1/3}$. It is seen that $H_s/\Delta D_{n50}$ equals the stability number $N_s = (K_D \cot \alpha)^{1/3}$, where K_D is the Hudson stability coefficient and α is the slope angle. (Note that the influence of the wave period is lacking in the parameter.)

According to extensive testing of rock slopes in head-on waves by DHI (Pilarczyk and Van der Meer) the values of the dimensionless parameter can be related to various types of rock slopes as follows:

$$H_s/\Delta D_{n50}$$

1 – 3	Conventional breakwater layer, start of damage
2 – 5	Conventional breakwater layer, failure
3 – 7	Berm breakwaters
5 – 50	Rock beaches

In the present tests we have

$$\Delta = \left(\frac{\rho_s}{\rho} - 1\right) = 1.65$$

$$D_{n50} = \left(\frac{12.7}{2.65}\right)^{1/3} = 1.69 \text{ cm}$$

$$\Delta \cdot D_{n50} = 2.79 \text{ cm}$$

and consequently the range of tests corresponds to

$$3.5 < H_s/\Delta D_{n50} < 7.1$$

which is the interval considered for berm type breakwaters.

SCALE EFFECTS CONSIDERATIONS

Provided that the grading of the stones is not too wide, say $\frac{d_{85}}{d_{15}} < 3$, and provided that the amount of fine material cannot block the pores (which is usually the case if $\frac{d_{85}}{d_{15}} < 3$) it is relevant to define a Reynolds' number with the characteristic length D_{n50}

$$Re = \frac{D_{n50} \sqrt{gH_s}}{\nu}$$

ν is the kinematic viscosity = 10^{-6} m²/s at 20°C.

With $H_s = 0.10 - 0.20$ m we get

$$1.7 \cdot 10^4 < Re < 2.4 \cdot 10^4$$

Juul Jensen and Klinting analysed the scale effects and found that no significant viscous scale effect is to be expected if in the outer part of the structure $Re > 0.6 \cdot 10^4$. Van der Meer found no scale effects for rock slopes with characteristic stone size of 20 mm, which is approximately the stone size in the present tests. This is also the experience of the Hydraulics Laboratory at the University of Aalborg.

However, although it is believed that a viscous scale effect is present it will be either negligible or will cause the results (in terms of amount of damage) to be on the safe side.

TEST PROCEDURE

Stability of round head and trunk in head-on waves

The initial profile in each test was the one shown in Fig. 3.

The waves were recorded continuously throughout all the tests.

The breakwater profile was measured after $N = 3000$ waves in all tests and also after 6000 and 9000 waves in some tests.

Moreover, the characteristics of the stone movements were found from video recordings of the movements of coloured stones.

Stability of trunk and longshore transport in oblique waves

In a trial test series it was found that for a given H_s , T_p the dynamically stable profiles in oblique waves within the tested range $\alpha \leq 30^\circ$, cf. Fig. 3, were almost identical to the profiles in head-on waves.

Thus in every test with oblique waves the initial profile was chosen as the one found after 3000 head-on waves.

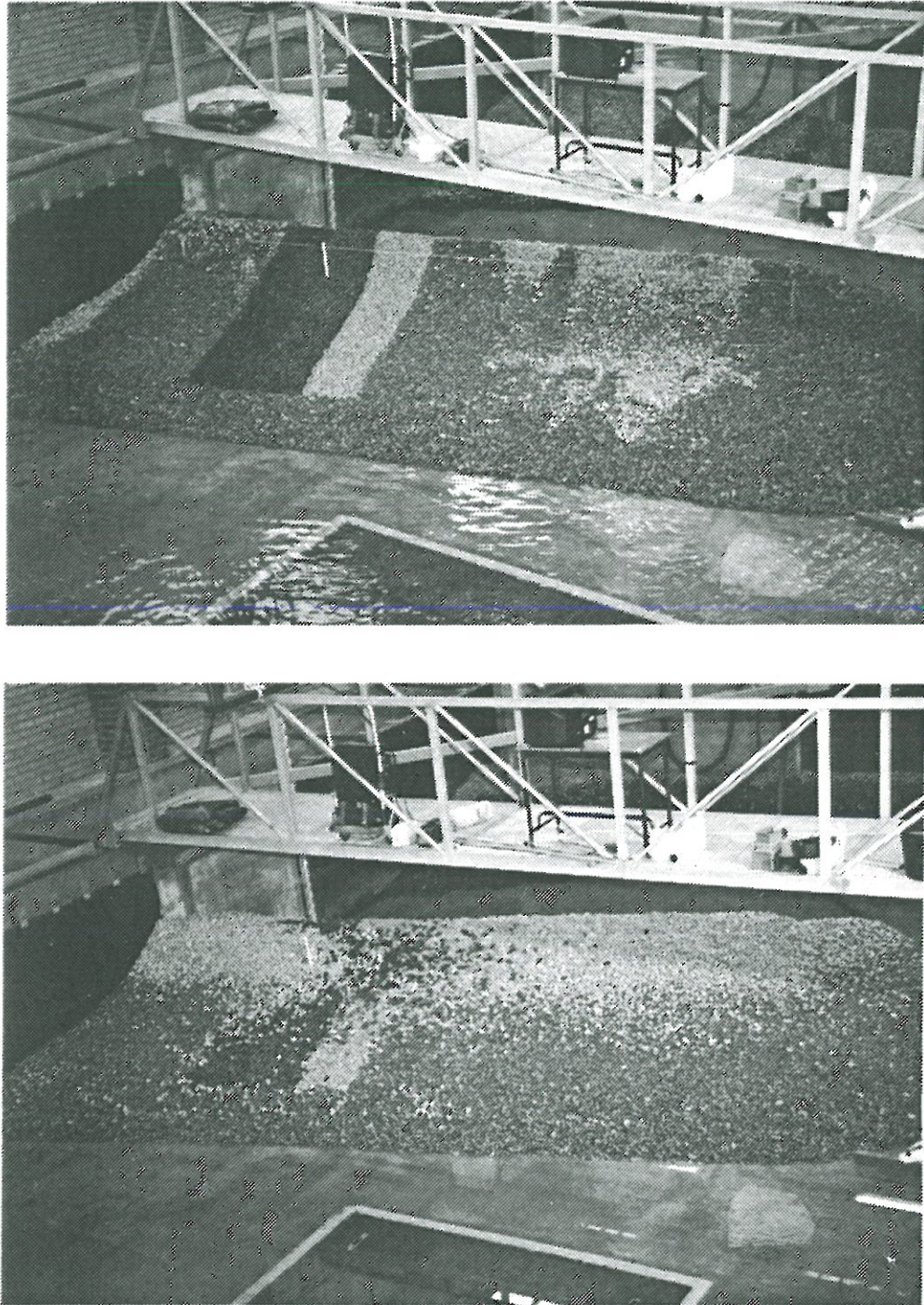


Fig. 5. Example of the breakwater before and after longshore transport tests in oblique waves.

The longshore transport was found from video recordings of the movements of coloured stones placed in three bands over the profile. Moreover, after a specific number of waves (or time) the number, the positions and the total weight of each type of coloured stones were recorded.

The band width and the number of waves N were adjusted to the sea state in such a way that within the test period the non-coloured stones upstream the coloured bands did not pass the downstream coloured band. In this way the average transport per second (or per wave) through a cross section could be found. Moreover, by studying the distribution of the coloured stones over the profiles the maximum erosion depth (i.e. the number of stone layers within which displacements take place) could be estimated.

Fig. 5 shows photos before and after a test.

TEST RESULTS

Profiles in head-on waves

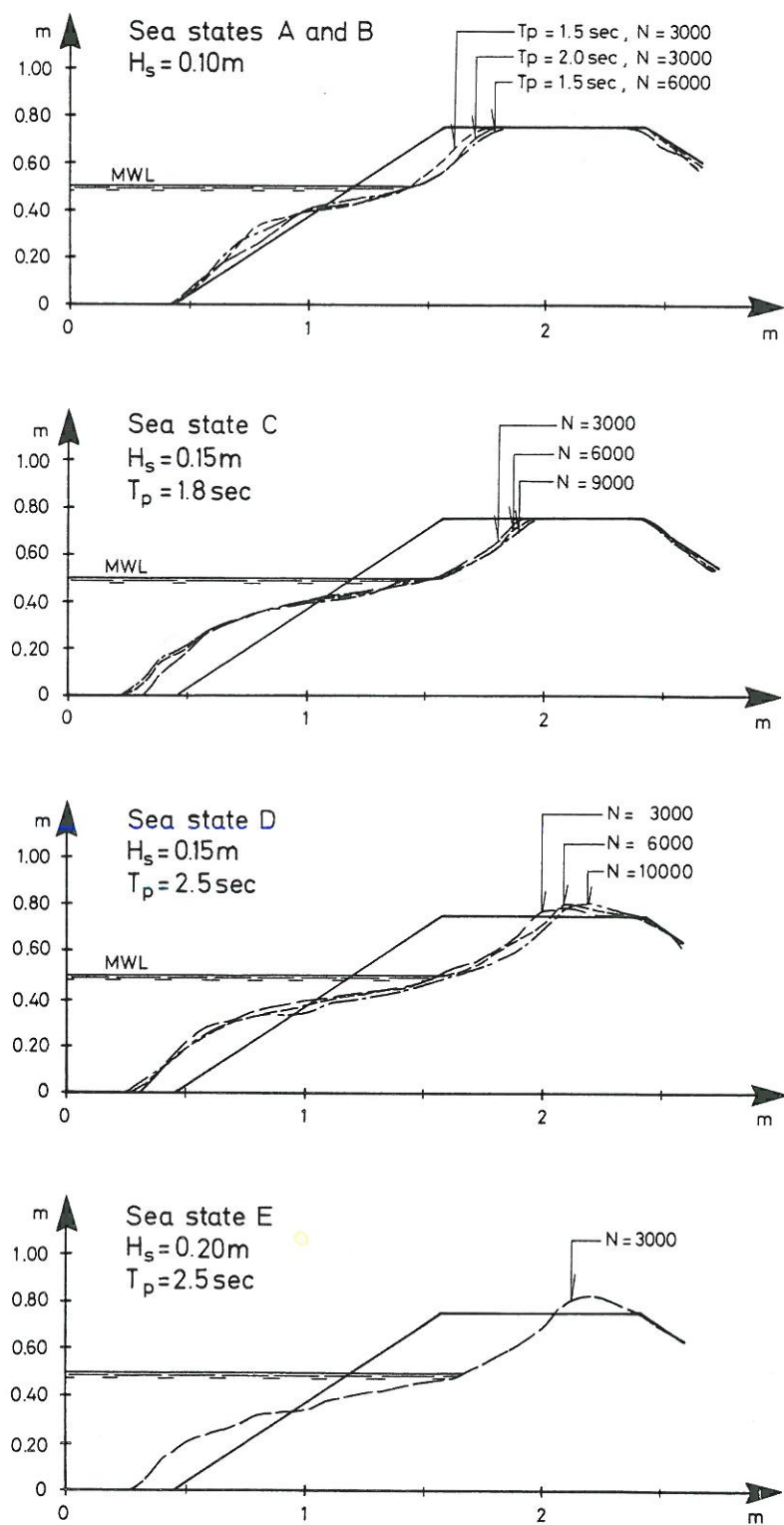


Fig. 6. Profiles in head-on waves.

Fig. 7 shows the various profiles for $N = 3000$.

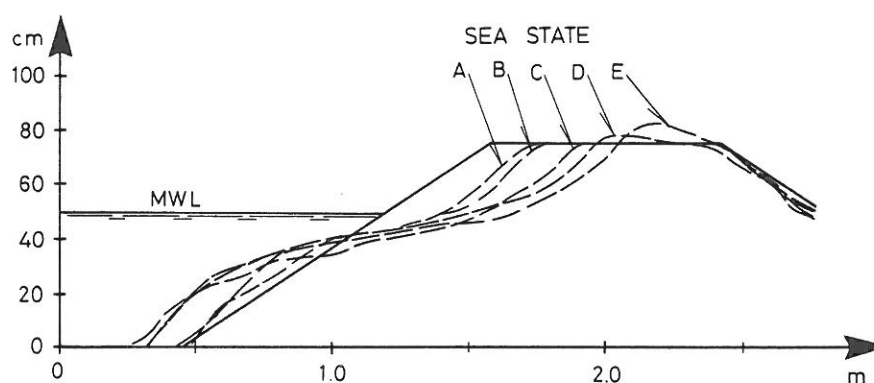


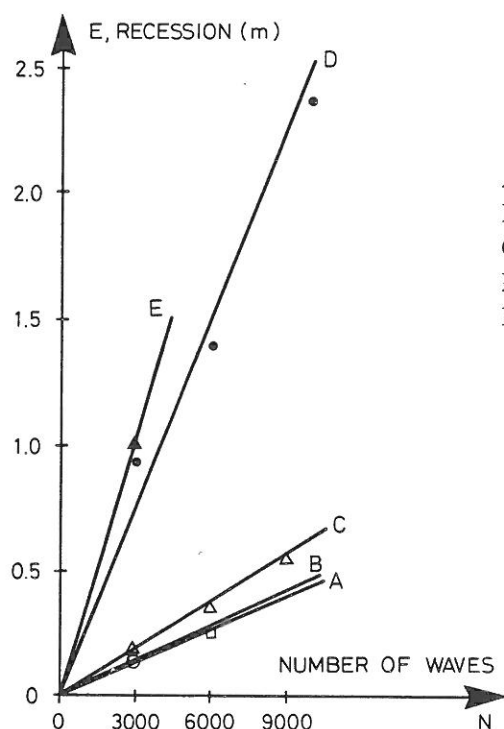
Fig. 7. Comparison of profiles after 3000 waves.

The material deficit is due to settlements caused by wave compaction and material transport across the crest.

Stability of the breakwater roundhead

The erosion of the roundhead is expressed in terms of the rate of recession of the crest measured along a longitudinal line parallel to the centerline of the breakwater, see Fig. 9.

Fig. 8 shows the recession as function of the number of waves. It is seen that the rate is almost constant for a certain sea state, i.e. a linear relationship between the recession of the crest end and the time (or number of waves).



	H_s (m)	T_p (s)	Rate of recession		$H_s/\Delta D_{n50}$
			m/1000 waves	m/h	
A	0.10	1.5	0.042	0.14	3.5
B	0.10	2.0	0.047	0.12	3.5
C	0.15	1.8	0.063	0.18	5.38
D	0.15	2.5	0.25	0.50	5.38
E	0.20	2.5	0.37	0.75	7.1

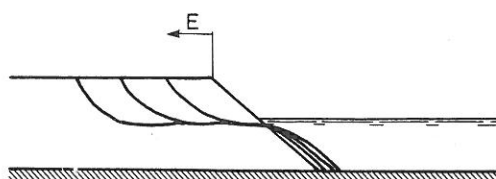


Fig. 8. Recession (erosion) rate of the roundhead.

It is seen from Fig. 8 that the roundhead erosion rate is small up to a certain sea state. When this is exceeded erosion is fast and the sea state seems to be characterized by the ability of practically every one of the waves to erode some of the stones from the roundhead and to displace (shift) them all the way across the roundhead.

Fig. 9 shows examples of the eroded longitudinal profiles as well as the bathymetry of the roundhead. A characteristic banana shape more pronounced than shown in the figure develops as the erosion proceeds.

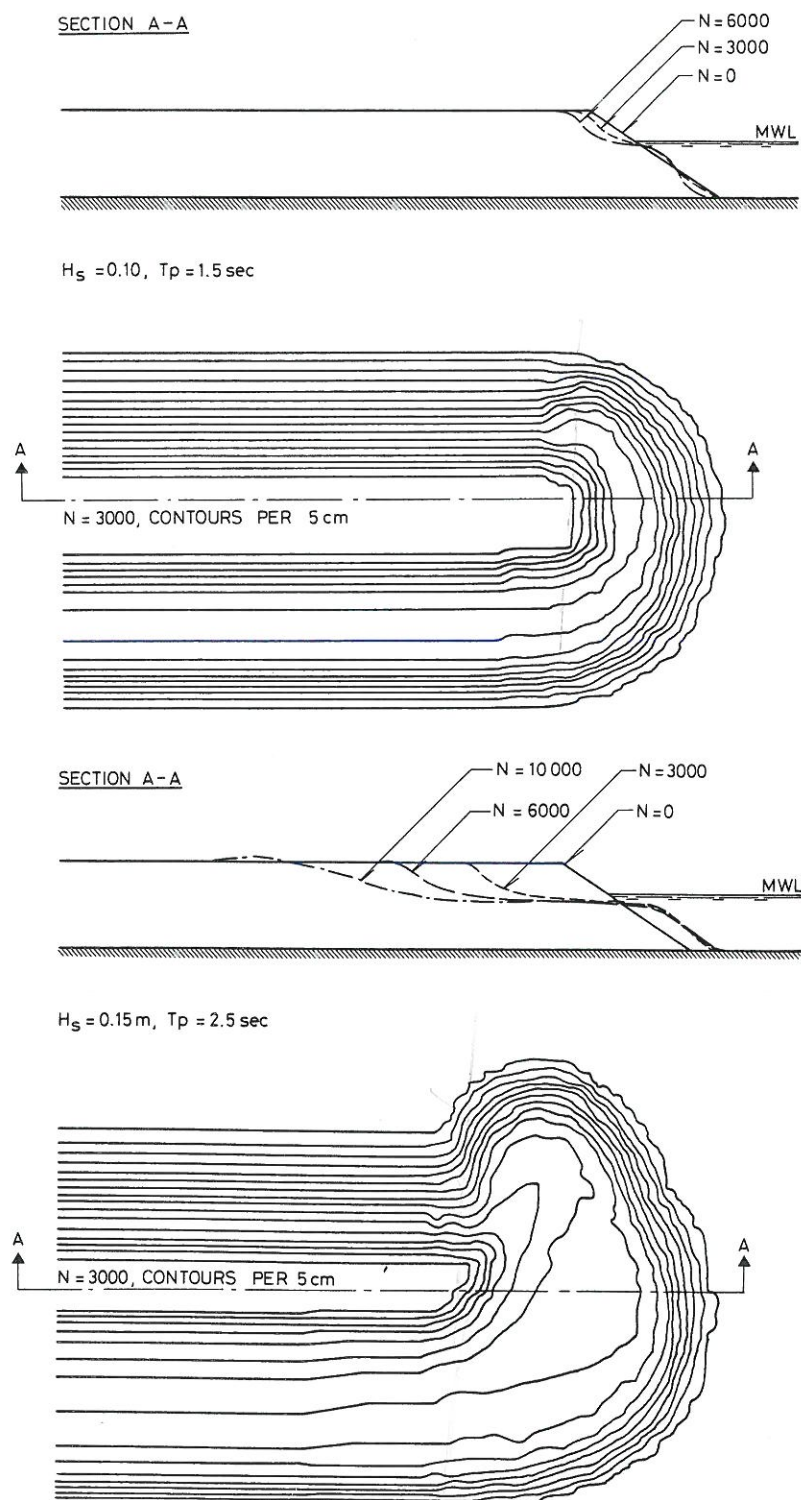


Fig. 9. Examples of erosion of roundhead.

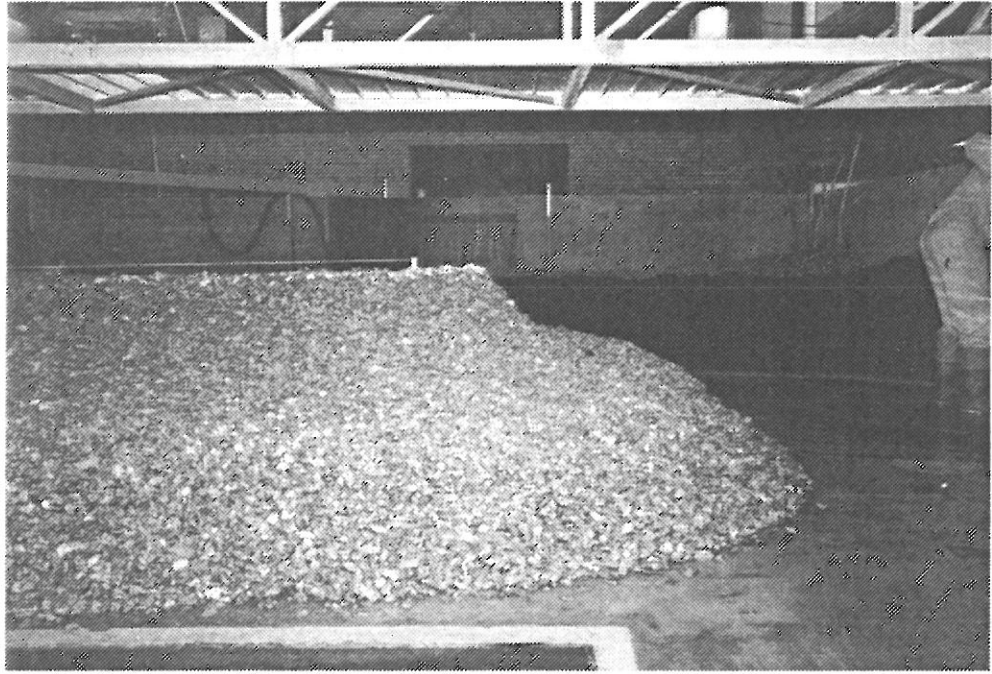


Fig. 10. Photo of roundhead after exposure to 3000 waves characterized by $H_s = 0.10$ m and $T_p = 2.0$ s.



Fig. 11. Photo of the roundhead after exposure to 3000 waves characterized by $H_s = 0.15$ m and $T_p = 2.5$ s.

Fig. 8 shows the measured recession, E , of the model as function of the number of waves. In order to make a simple non-dimensional representation for the recession a "Shields approach" is used for the examination of the governing parameters.

The sediment transport close to the bed is usually described in a dimensionless form by the Meyer-Peter formula

$$8(\theta - \theta_c)^{3/2} = \frac{q_b}{\sqrt{\Delta g \cdot d^3}}$$

where

$$\theta = \frac{\tau}{\rho \cdot \Delta \cdot g \cdot d} \text{ is the dimensionless bed shear}$$

$$q_b = \text{transport}$$

$$\Delta = \rho_s / \rho - 1, \quad \rho_s = \text{density of stone}$$

$$\rho = \text{density of water}$$

$$d = \text{average grain size}$$

$$g = \text{gravity}$$

$$\theta_c = \text{critical dimensionless bed shear stress}$$

$$\tau = \text{shear stress}$$

The bed shear stress τ can be expressed as a function of the flow velocity amplitude near the bed.

$$\tau = \frac{1}{2} f_w \rho \cdot u^2$$

$$f_w = \text{Jonsons wave friction factor}$$

$$u = \text{amplitude of flow velocity}$$

Considering the shallow water in the breaker zone on the slope the flow velocity is assumed to be proportional to $\sqrt{g \cdot H_s}$.

In the Meyer-Peter formula the bed transport is related to $\sqrt{\Delta \cdot g \cdot d^3}$ in order to get a dimensionless expression for the bed transport. In the present experiments a formula for the erosion of the roundhead in terms of the recession E (of Fig. 8) is sought. Dividing the recession E with the deep water wavelength $\lambda \cong g \cdot T^2$ a dimensionless recession is found.

Applying in principle the Meyer-Peter formula we obtain

$$\frac{E}{\lambda} \cong (\theta - \theta_c)^{3/2}$$

As a first simplified approach the bed transport (erosion) is related to the characteristic sea state parameters H_s and T_z

$$\frac{E}{g \cdot T_z^2} \cong \left(\frac{\frac{1}{2} f_\omega \rho \cdot g \cdot H_s}{\rho \cdot \Delta \cdot g \cdot D_{n50}} - \theta_c \right)^{3/2}$$

$$E \cong \frac{T_z^2 \cdot g (H_s - \Delta \cdot D_{n50} \cdot \theta')^{3/2}}{\Delta^{3/2} \cdot D_{n50}^{3/2}}$$

The last expression is based on the simplifying assumption that Jonson's wave friction factor f_ω can be taken as constant. f_ω may be calculated using for example Peter Nielsens expressions or Bijkers expressions. However, because the flow over the bed is composed of oscillatory flow and current, and because the flow is highly influenced by the wave breaking it is rather complicated to determine a meaningful f_ω .

f_ω is a function of a/k , i.e. the water particle displacement divided by the bed roughness, but the variation is weak. In a normal breakwater situation a/k will probably show small variations and therefore the variation of f_ω will be marginal.

By applying a least square fit the threshold value θ' is found to be 0.82. This value may look small compared to a threshold value for stone movements estimated from the Hudson formula $H_s = (K_D \cdot \cot \alpha)^{1/3} \cdot \Delta \cdot D_{n50}$. However, it has to be remembered that the threshold value is related to a characteristic wave height, H_s , which is an over-simplification as it does not reflect correctly the effect of an irregular wave train. Fig. 12 shows the dimensionless recession.

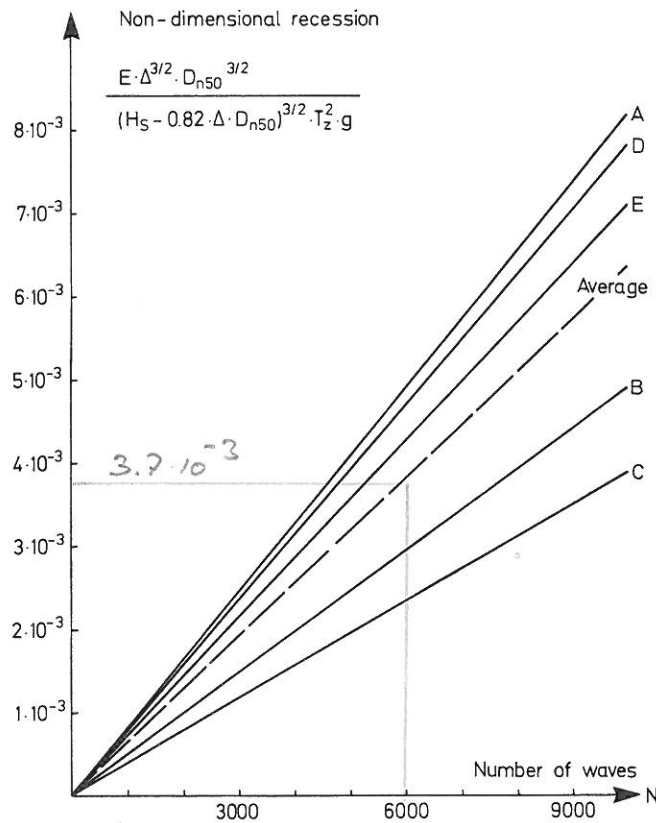


Fig. 12. Non-dimensional recession (erosion) of the roundhead).

It is seen that a completely satisfactory universal representation is not obtained. However, until more tests and further analyses are completed Fig. 12 might provide a crude first estimate of roundhead recessions.

In order to utilize the test results for a crude estimation of roundhead erosion also for breakwaters with a geometry somewhat different from the one presented here it is suggested to consider the resistance per unit volume stone independent on the cross sectional geometry (of course within some limits).

Having observed in the tests that the erosion reached a level of approximately $H_s/2$ below the still water level the dimensionless cross section area $A^* = A/(\Delta \cdot D_{n50})^2$ is introduced, where A is defined in Fig. 13.

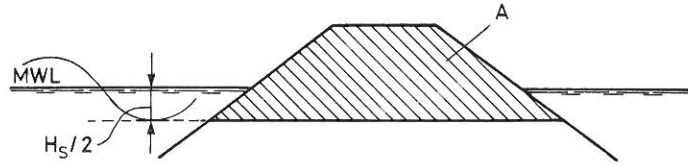


Fig. 13. Definition of eroded cross sectional area, A .

The prototype recession E_p of some structure might then be expressed in terms of:

- E_{AU} — the recession found using Fig. 12 (corresponding to a geometry similar to the Aalborg University (AU) model)
- A_{AU}^* — dimensionless cross sectional area of test geometry
- A_p^* — dimensionless cross sectional area of prototype

$$E_p = \frac{A_{AU}^*}{A_p^*} \cdot E_{AU}$$

$$\cong \frac{450 \cdot E_{AU}}{A_p^*}$$

Based on the presented model tests and the behaviour of some prototype breakwaters the following somewhat premature recommendations valid for permanent designs are proposed:

	$H_s/\Delta D_{n50}$
For trunks exposed to steep oblique waves	< 4.5
For trunks exposed to long oblique waves	< 3.5
For roundheads	< 3

These values should be used as guidelines *only* if no other more qualified information is available. This is because the parameter $H_s/\Delta D_{n50}$ is insufficient as it among other things do not contain the effect of wave length and the effect of the duration of the sea.

It is believed that in additional investigations of erosion of reshaping breakwaters it will be necessary in principle to examine and summarize the respons from every single wave instead of using characteristic parameters like H_s to characterize the sea state. This is because the character of the flow kinematics in the erosion zones are strongly dependend on the size and the steepness of the single waves.

Effect of oblique waves on the trunk

The steady state transport of stones along the trunk was studied for two angles of wave attack, $\alpha = 15^\circ$ and 30° , cf. Fig. 3.

The results are given in Table 1.

Table 1. Steady state mass transport along the trunk.

Test no	Sea state	H_s cm	T_p sec.	α deg.	Average mass transport, Q	
					g/wave	g/sec.
6	A	10	1.5	15	0.48	0.45
7	B	10	2.0	—	0.95	0.68
8	C	15	1.8	—	14.7	11.7
9	D	15	2.5	—	12.4	7.1
10	E	20	2.5	—	56.0	32.0
11	A	10	1.5	30	0.78	0.74
12	B	10	2.0	—	1.29	0.92
13	C	15	1.8	—	20.5	16.3
14	D	15	2.5	—	30.7	17.5
15	E	20	2.5	—	62.5	35.7

It is seen that the sea states A and B only cause very small mass transports corresponding more or less to the onset of long-structure transport. It is also seen that the more severe sea states result in a significant transport which in typical prototype situations will change the geometry of the structure and might endanger the stability.

The results look homogeneous in the sense that the transport increases with wave height, wave period and angle of incidence in the tested ranges. The exception is test no 9 where the transport seems to be unexpected low.

The following formula for longshore transport on gravel beaches was given by van Hijum et al. 1982:

$$\frac{S}{g D_{90}^2 T_s} = 7.12 \cdot 10^{-4} \frac{H_{sd} \cos^{1/2} \alpha}{D_{90}} \left(\frac{H_{sd} \cos^{1/2} \alpha}{D_{90}} - 8.3 \right) \frac{\sin \alpha}{\tanh (k_s h)_v}$$

where S longshore transport in m^3/s
 D_{90} grain diameter corresponding to 90% passing (by weight)
 T_s 15% excess value of wave period
 H_{sd} deep water significant wave height
 α angle of incidence for the waves
 $k_s = \frac{2\pi}{L_s}$ where L_s is the 15% excess value of wave length
 h depth of foreshore
 $(k_s h)_v$ (kh) on the foreshore

A comparison with this formula is not possible because it predicts zero transport for all the test conditions except for test no 10 in which case the formula predicts the measured transport with good accuracy. One reason why the formula is not generally applicable is probably that it is based on tests with H_s/D_{n50} values outside the berm breakwater range. The implemented empirical threshold value for the onset of longshore transport ($H_{sd} \cos^{1/2} \alpha / D_{90} = 8.3$) does not correspond to the observations in the Aalborg University tests.

A study of a general parametric representation of the test results is in progress. However, it is believed that more tests will be necessary to confirm the validity of any parametric representation of the long structure mass transport.

CONCLUSIONS

The roundhead erosion and the erosion of the trunk in oblique waves have a very strong non-linear dependency on the sea state. Below a certain sea state threshold value the erosion rates are very small, but excess of this value causes a drastic increase in the erosion. Consequently identification and consideration of this threshold value are of great importance in the design process.

ACKNOWLEDGEMENT

The help of Mr. Van der Meer in estimating cross section profiles during the planning of the tests is gratefully acknowledged.

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